

LIFE-CYCLE ROBUSTNESS OF DETERIORATING CONCRETE STRUCTURES

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ABSTRACT

The life-cycle structural robustness is generally investigated with respect to the initial time of construction, when the structure is intact. However, in the evaluation of life-cycle robustness of existing structures it may be of interest to evaluate the impact of deterioration on the structural performance with reference to the actual damage state and performance level existing at the time of inspection and/or assessment. In this paper, a robustness criterion presented in previous works is extended to consider the influence of the damage level on the time-variant robustness of existing concrete structures over the remaining structural lifetime. In addition, the role of the importance of the structure in the assessment of the life-cycle robustness is considered. The effectiveness of the proposed approach is shown through the application to the deterministic and probabilistic assessment of the life-cycle robustness of a reinforced concrete pier of an existing bridge.

KEYWORDS

Structural robustness, Life-cycle performance, Concrete structures, Bridges, Corrosion.

INTRODUCTION

As a consequence of several dramatic structural failures, due to extreme hazards such as blast or aircraft impact, the importance of reliable design procedures leading to conceive robust structures is nowadays widely recognized (Ellingwood and Dusenberry 2005, Ellingwood 2006, Ghosn and Frangopol 2007). This is not limited to buildings, but it is a major concern also for bridges (Starossek 2008). Therefore, new design concepts and methods are needed to ensure safety, redundancy and robustness of buildings and bridges against the occurrence of exceptional damaging events (Ghosn *et al.* 2010, Okasha and Frangopol 2010a, Zhu and Frangopol 2012, 2013, 2014, 2015, Frangopol and Saydam 2014). In addition, damage involving disproportionate effects could also arise continuously in time, due to aging and deterioration processes (Frangopol and Curley 1987, Biondini and Restelli 2008, Biondini *et al.* 2008, Biondini 2009, Okasha and Frangopol 2010b, Decò *et al.* 2011, Biondini and Frangopol 2014a, 2014b). These effects are particularly relevant for bridge structures due to their environmental exposure. Notable events of bridge collapses due to the environmental aggressiveness and related phenomena, such as corrosion and fatigue, include the Silver Bridge in 1967, and the Mianus River Bridge in 1983. Structural robustness should therefore be considered as key factor for a rational approach to life-cycle design of deteriorating structures (Biondini and Frangopol 2014a). In this context, it is of great interest to investigate the evolution in time of robustness under a progressive deterioration of the structural performance.

Recently, the time factor has been explicitly included in a lifetime scale for a time-variant measure of structural robustness both in deterministic and probabilistic terms (Biondini 2009, Biondini and Frangopol 2014a). A robustness criterion has been introduced by comparing the loss of performance due to a certain damage scenario with an acceptable robustness target. Moreover, the life-cycle structural robustness has been investigated with respect to the initial time of construction, when the structure is intact. However, in the evaluation of life-cycle robustness of existing structures it may be of interest to evaluate the impact of deterioration on the structural performance with reference to the actual damage state and performance level existing at the time of inspection and/or assessment. In this paper, the proposed robustness criterion is hence extended to consider the influence of the damage level on the time-variant robustness of existing structures over the remaining structural lifetime, with emphasis on concrete structures exposed to corrosion. In addition, the importance of the structure in the assessment of the life-cycle robustness is considered.

To this purpose, criteria and methods for the definition of time-variant performance indicators and quantitative evaluation of life-cycle robustness of concrete structures are presented. The effects of the damage process on the structural performance are evaluated by using a methodology for life-cycle assessment of concrete structures in aggressive environment under uncertainty (Biondini *et al.* 2004, 2006). The proposed approach is applied to the deterministic and probabilistic assessment of the life-cycle robustness of a reinforced concrete (RC) bridge pier with box cross-section by taking into account the actual damage state and performance level at different time instants over the structural lifetime. The results highlight the essential role of the environmental exposure and show the influence of the importance of the structure in the assessment of the life-cycle structural robustness.

TIME-VARIANT MEASURE OF STRUCTURAL PERFORMANCE AND DAMAGE

Performance Index

A failure of a system is generally associated with the violation of one or more limit states. Focusing on concrete structures, limit states of interest are the occurrence at the material level of local failures associated to cracking of concrete and yielding of steel reinforcement, which represent warnings for initiation of damage propagation, as well as reaching of global failures associated with the ultimate capacity of critical cross-sections of structural members and/or system collapse. Since the structural performance of concrete structures deteriorates over time, the limit states need to be evaluated by means of time-variant structural analyses taking into account the effects of the damage process (Biondini *et al.* 2004, 2006).

By denoting $\lambda = \lambda(t) \geq 0$ a time-variant performance indicator associated to the occurrence of a prescribed limit state, its ratio to the performance indicator $\lambda_0 = \lambda(t_0) \geq \lambda(t)$ referred to a time instant $t_0 \leq t$ provides an effective time-variant measure of structural performance within the range $[0, 1]$ over the time interval $[t_0, t]$:

$$\rho(t, t_0) = \frac{\lambda(t)}{\lambda(t_0)} \quad (1)$$

In general, the reference time is associated with the initial time of construction, $t_0=0$, when the structure is intact. However, in the evaluation of life-cycle robustness of existing structures it may be of interest to evaluate the expected impact of deterioration on the structural performance at time t with reference to the actual damage state and performance level existing at time $t_0 > 0$.

Damage Index

In concrete structures damage is generally induced by diffusion of aggressive agents, such as sulphates and chlorides, which may lead to deterioration of concrete and corrosion of reinforcement (CEB 1992). The diffusion process can be effectively described by using the Fick's diffusion equation (Glicksman 2000):

$$D \nabla^2 C = \frac{\partial C}{\partial t} \quad (2)$$

where D is the diffusivity coefficient of the medium, $C=C(\mathbf{x}, t)$ is the concentration of the chemical component at point \mathbf{x} and time t , $\nabla C = \mathbf{grad} C(\mathbf{x}, t)$ and $\nabla^2 = \nabla \cdot \nabla$. In this study, such equation is solved numerically by using cellular automata (Biondini *et al.* 2004).

Structural damage induced by diffusion is modelled by a degradation law of the effective resistant area for both concrete and reinforcing steel bars by means of dimensionless damage indices $\delta_c = \delta_c(t)$ and $\delta_s = \delta_s(t)$, respectively, which provide a direct measure of the damage level within the range $[0, 1]$. In particular, damage rates depend on the concentration of the aggressive agent (Bertolini 2008). Despite the complexity of such relationship at the microscopic level, simple coupling models can often be successfully adopted at the macroscopic level in order to reliably predict the time evolution of structural performance (Biondini *et al.* 2004, 2006). In this study, the damage indices $\delta_c = \delta_c(\mathbf{x}, t)$ and $\delta_s = \delta_s(\mathbf{x}, t)$ at point \mathbf{x} and time t are correlated to the diffusion process as follows:

$$\frac{\partial \delta_c(\mathbf{x}, t)}{\partial t} = \frac{C(\mathbf{x}, t)}{C_c \Delta t_c} \quad (3)$$

$$\frac{\partial \delta_s(\mathbf{x}, t)}{\partial t} = \frac{C(\mathbf{x}, t)}{C_s \Delta t_s} \quad (4)$$

where C_c and C_s represent the values of constant concentration $C(\mathbf{x},t)$ which lead to a complete damage of the materials after the time periods Δt_c and Δt_s , respectively. The damage rate coefficients $q_c=(C_c\Delta t_c)^{-1}$ and $q_s=(C_s\Delta t_s)^{-1}$ depend on both the type of corrosion mechanism and corrosion penetration rate. Moreover, the initial conditions $\delta_c(\mathbf{x},t_{cr})=\delta_s(\mathbf{x},t_{cr})=0$ with $t_{cr}=\min\{t \mid C(\mathbf{x},t)\geq C_{cr}\}$ are assumed, where t_{cr} is the corrosion initiation time and C_{cr} is a critical threshold of concentration (Biondini *et al.* 2004). These relationships can be calibrated based on available data for corrosion rate under sulphate and chloride attacks (Pastore and Pedferri 1994, Bertolini *et al.* 2004).

The damage indices δ_c and δ_s provide a comprehensive description of the damage evolution over the structure. However, due to their *local* nature, they do not seem handy for global evaluations of system robustness. A more synthetic *global* measure of damage is necessary. A global damage index Δ^* within the range $[0, 1]$ may be derived from δ_c and δ_s by a weighted average over the volume of the materials (Biondini 2009). By denoting $\Delta_c=\Delta_c(t)$ and $\Delta_s=\Delta_s(t)$ the contribution of concrete and steel, respectively, for a concrete member the time-variant global damage index $\Delta^*=\Delta^*(t)$ can be defined at the cross-sectional level as follows:

$$\Delta^*(t) = [1 - \omega(t)]\Delta_c(t) + \omega(t)\Delta_s(t) \quad (5)$$

$$\Delta_c(t) = \frac{\int_{A_c} w_c(\mathbf{x},t) \delta_c(\mathbf{x},t) d\mathbf{x}}{\int_{A_c} w_c(\mathbf{x},t) d\mathbf{x}} \quad (6)$$

$$\Delta_s(t) = \frac{\sum_m w_{sm}(t) \delta_{sm}(t) A_{sm}}{\sum_m w_{sm}(t) A_{sm}} \quad (7)$$

where $\omega=\omega(t)$, $w_c=w_c(\mathbf{x},t)$, and $w_{sm}=w_{sm}(\mathbf{x}_m,t)$ are suitable weight functions, A_c is the area of the concrete matrix, and A_{sm} is the area of the m^{th} steel bar located at $\mathbf{x}_m=(y_m, z_m)$. It is worth noting that this cross-sectional formulation can be extended at the structural level by an average integration over all members of the system. In case any portion of material volume is expected to play a specific role in the damage process, suggested values for the weights are $w_c=w_{sm}=1$ and $\omega=(f_s A_s)/(f_c A_c)$, where ω is the mechanical ratio of reinforcement, f_c is the concrete strength in compression, f_s is the steel strength, and $A_s=\sum_m A_{sm}$.

For existing structures it would be of interest to evaluate the damage $\Delta^*=\Delta^*(t)$ expected at time t with respect to the actual damage state $\Delta^*=\Delta^*(t_0)$ existing at current time $t_0>0$. To this purpose, the following damage index provides a time-variant measure of damage within the range $[0, 1]$ over the time interval $[t_0, t]$:

$$\Delta(t, t_0) = \frac{\Delta^*(t) - \Delta^*(t_0)}{1 - \Delta^*(t_0)} \quad (8)$$

where $\Delta^*(t_0)=0$ and $\Delta(t, t_0)=\Delta^*(t)$ for a reference time $t_0=0$ associated with the initial construction.

TIME-VARIANT STRUCTURAL ROBUSTNESS

Robustness Criterion

The knowledge of the time-variant performance index $\rho=\rho(t, t_0)$ is in general not sufficient to formulate a measure of structural robustness. In fact, structural robustness can be viewed as the ability of the system to suffer an amount of damage not disproportionate with respect to the causes of the damage itself (Ellingwood and Dusenberry 2005). According to this definition, a measure of robustness should arise by comparing the system performance in the original state, in which the structure is fully intact, and in a perturbed state, in which a prescribed damage scenario is applied (Frangopol and Curley 1987, Biondini and Restelli 2008). To this aim, a robustness criterion is formulated as a function of both the performance index $\rho=\rho(t, t_0)$ and the related damage index $\Delta=\Delta(t, t_0)$ as follows:

$$R(t, t_0) = \rho(t, t_0)^\alpha + \Delta(t, t_0)^\alpha \quad (9)$$

where $R=R(\rho, \Delta)=R(t, t_0)$ is a robustness factor, and α is a shape parameter of the boundary $R=1$. The structural system is robust when the criterion is satisfied ($R\geq 1$), and not robust otherwise ($R<1$). This criterion has been proposed in Biondini (2009) for a measure of time-variant robustness with respect to the initial undamaged state of the structure ($t_0=0$) and is here extended to the case of damaged structures ($t_0>0$).

Importance Factor

The value of the parameter α should be selected according to the acceptable level of damage susceptibility for the structure under investigation. A value $\alpha=1$, which indicates a proportionality between acceptable loss of performance and damage, should be appropriate in most cases. Values $\alpha>1$ could be required for structures of strategic importance, and values $\alpha<1$ should be avoided, since they allow for disproportionate damage effects, or used for temporary structures. The importance factor α emphasizes that the robustness measure depends not only on system properties and damage mechanisms, but also on the importance of the system.

A proper value of the importance factor α can be chosen with reference to the area $A=A(\alpha)\in[0,1]$ of the region lying below the boundary curve $R=1$ (Di Silvestri *et al.* 2014):

$$A(\alpha) = \left\{ \int_0^1 \rho(\alpha, \Delta) d\Delta \mid R=1 \right\} = \int_0^1 (1 - \Delta^\alpha)^{\frac{1}{\alpha}} d\Delta \quad (10)$$

which leads to $A=0$ for $\alpha=0$, $A=0.5$ for $\alpha=1$, and $A=1$ for $\alpha=\infty$. This concept is illustrated in Figure 1 for (a) $\alpha=1$ and (b) $\alpha>1$. Possible values of the importance factor α and the corresponding area $A(\alpha)$ for temporary structures, ordinary structures, and structures of strategic importance, are listed in Table 1.

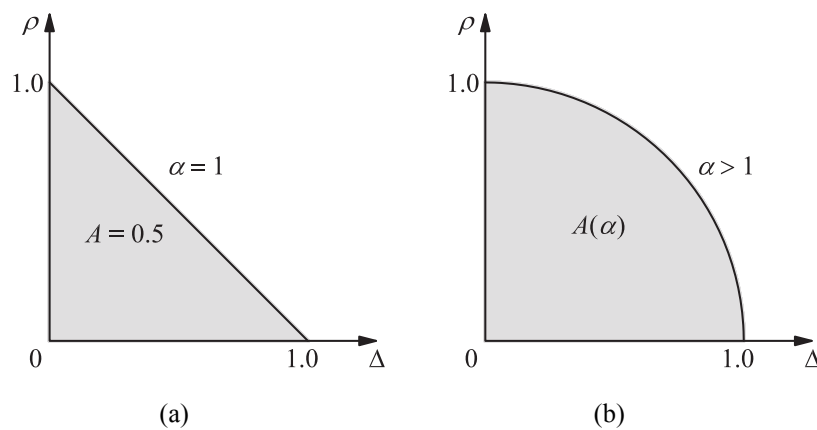


Figure 1. Area $A=A(\alpha)$ of the region lying below the boundary curve $R=1$: (a) $\alpha=1$ and (b) $\alpha>1$.

Table 1. Importance factor α for different type of structures.

Type of Structure	Area $A(\alpha)$	α
Temporary	0.25	0.6
Ordinary	0.50	1.0
Strategic	0.75	1.8

APPLICATION TO A RC BRIDGE PIER

The life-cycle reliability and maintenance planning of the RC piers of an existing bridge have been investigated in a previous work (Biondini *et al.* 2006). The life-cycle structural robustness of the piers has been also evaluated with reference to the undamaged state at the initial time $t_0=0$ (Biondini and Frangopol 2012). In this study, the robustness analysis is extended to consider the impact of deterioration on the structural performance with reference to the actual damage state and performance level existing at current time $t_0>0$. In addition, the role of the importance of the structure in the assessment of the life-cycle robustness is considered.

The box-cross section of the bridge pier is shown in Figure 2.a. The cross-section has main geometrical dimensions $d_y=8.20$ m and $d_z=9.00$ m, and it is reinforced with $160+248=498$ steel bars having diameter $\varnothing=18$ mm and $\varnothing=30$ mm, respectively, as shown in Figure 2.b. The material strengths are $f_c=30$ MPa for concrete in compression, and $f_{sy}=500$ MPa for reinforcing steel. Additional information on the constitutive properties of the materials and the methodology adopted for the cross-sectional non-linear structural analysis can be found in Biondini *et al.* (2006) and Biondini and Frangopol (2012).

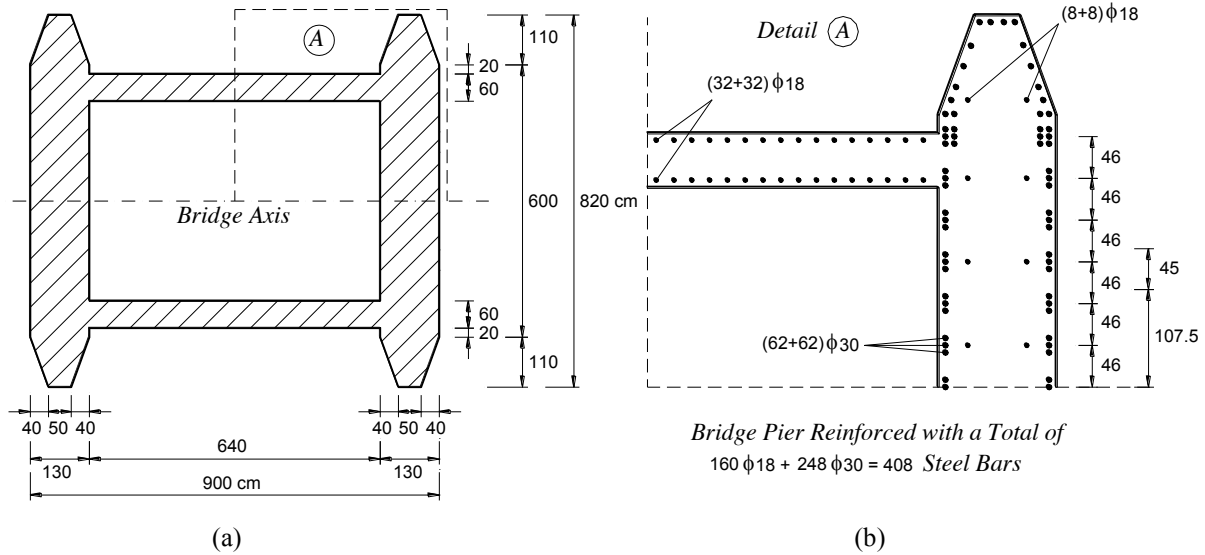


Figure 2. Box cross-section of a bridge pier. (a) Geometrical dimensions. (b) Detail with reinforcement layout.

A diffusivity coefficient $D = 10^{-11} \text{ m}^2/\text{sec}$ is assumed. Damage rates are defined by assuming $C_{cr}=0$, $C_c=C_s=C_0$, $\Delta t_c=5$ years and $\Delta t_s=7.5$ years. Figure 3.a shows the grid of the cellular automaton adopted for the simulation of the diffusion process and the location of the aggressive agent, with concentration $C(t)=C_{ext}$ along the external surface of the pier and $C(t)=C_{int}$ along the internal one. As an example, Figure 3.b shows the map of concentration $C(\mathbf{x},t)/C_0$ of the aggressive agent for the case $C_{ext}=2C_{int}=C_0$ after 50 years from the initial time of diffusion penetration.

Table 1. Concentration of the agent for the exposure scenarios.

	Case (I)	Case (II)	Case (III)
C_{ext}	C_0	C_0	0
C_{int}	C_0	0	C_0

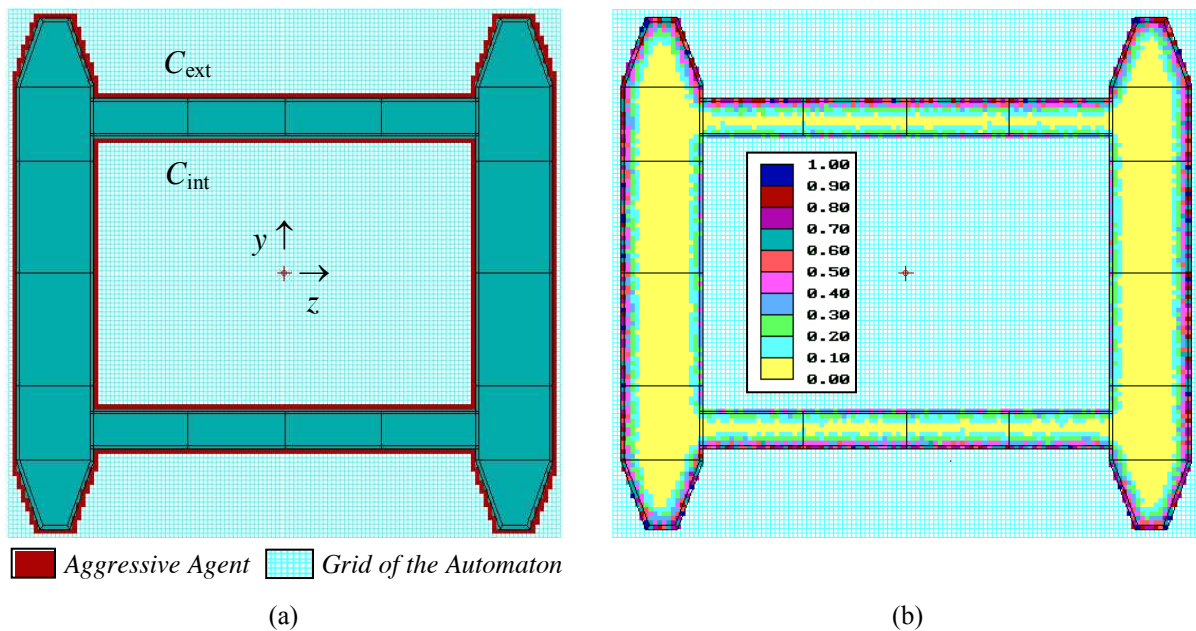


Figure 3. Box cross-section of a bridge pier. (a) Structural model, grid of the cellular automaton and location of the aggressive agent. (b) Map of the concentration $C(\mathbf{x},t)/C_0$ of the aggressive agent for $C_{ext}=2C_{int}=C_0$ after 50 years from the initial time of diffusion penetration.

The effects of damage induced by diffusion over a 50-year lifetime is shown in Figure 4.a in terms of time-evolution of the resistant value $M_R=M_R(t)$ of the bending moment M_z under the axial force $N=-100$ MN for the

three exposure scenarios listed in Table 1. The corresponding time evolution of the damage index $\Delta=\Delta(t,t_0)$ is shown in Figure 4.b for $t_0=0$, that is $\Delta(t,0)=\Delta^*(t)$. The comparison of the results illustrated in Figures 4.a and 4.b shows that, as expected, case (I) with full exposure of the cross-section is the worst scenario in terms of global damage and strength deterioration. Moreover, case (II) with external exposure only is more critical than case (III) with internal exposure only, since the area exposed to the aggressive environment in case (II) is larger than in case (III). However, the separate knowledge of the time quantities M_R and Δ does not allow to measure structural robustness.

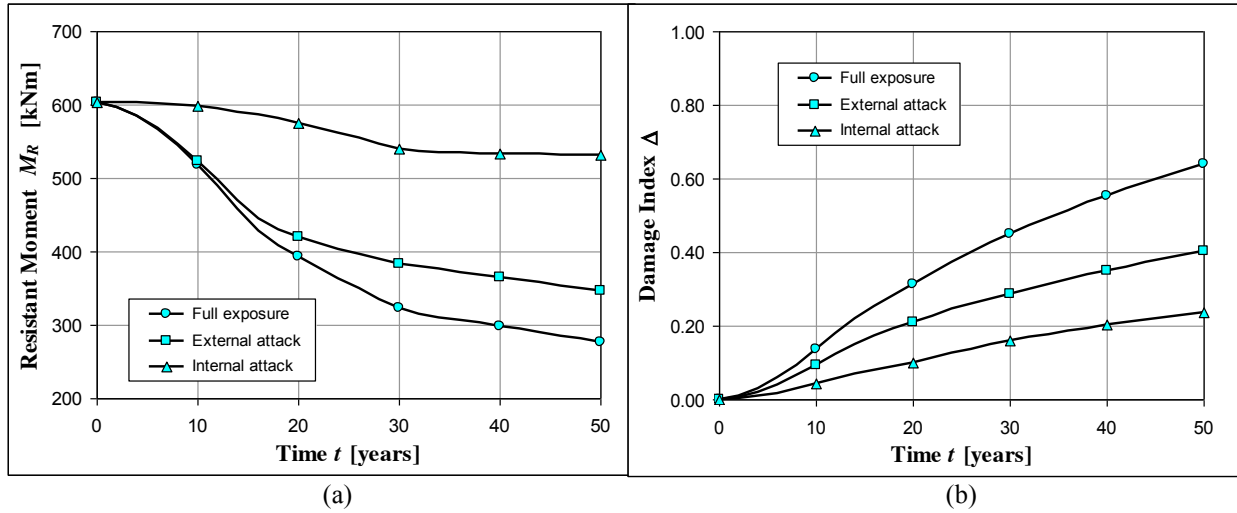


Figure 4. Time evolution of (a) resistant bending moment $M_R=M_R(t)$, and (b) damage index $\Delta=\Delta(t,t_0)$ for $t_0=0$ ($\Delta=\Delta^*$) under the three exposure damage scenarios listed in Table 1.

A time-variant measure of structural robustness is achieved by assuming the resistant bending moment as performance indicator, or $\lambda(t)=M_R(t)$, and by relating the corresponding performance index $\rho=\rho(t,t_0)$ to the damage index $\Delta=\Delta(t,t_0)$. The time evolution of the robustness factor $R=R(t,t_0)$ for the three investigated exposure scenarios is shown in Figure 5 with respect to three reference time instants $t_0=0, 10$, and 20 years, and for two values of the importance factor $\alpha=1.0$ (Figure 5.a) and $\alpha=1.8$ (Figure 5.b).

The diagrams shown in Figure 6.a are associated with an importance factor $\alpha=1.0$, that is appropriate for ordinary bridge structures. First of all it is noted that the robustness of the bridge pier may increase over time, despite the structural performance significantly decreases, with a reduction of the resistant bending moment of over 50% after 50 years of lifetime for the case of full exposure (Figure 5.a). In particular, for $t_0=0$ and $t_0=10$ years the bridge pier is robust for case (III) and not robust for case (II) over the whole lifetime. For case (I) the pier is not robust only in the early stage of the lifetime, since after about 20 years the susceptibility of the structure to damage tends to decrease, reaching $R\approx 1$ after about 40 years. However, this recovery of robustness is achieved after extensive damage which may involve an unacceptable decay of bending strength. It is also worth noting that in case (I) the lifetime robustness is always higher than in case (II), and for $t_0=20$ years it is also higher than in case (III), despite that a reversed tendency is observed in terms of resistant moment $M_R=M_R(t)$. This clearly indicates that strength and robustness are different performance indicators that may exhibit opposite trends over time.

Depending on location and traffic demand, the functionality of bridges could represent a key issue to ensure the resilience of infrastructure road and highway networks. Therefore, for bridges of strategic importance, values $\alpha>1$ should be adopted in the assessment of the life-cycle structural robustness. The diagrams shown in Figure 6.b are associated with an importance factor $\alpha=1.8$, that corresponds to $A(\alpha)=0.75$ (see Table 1). For this level of importance, the bridge pier is not robust over the 50-year lifetime for all cases studied. In some cases, the robustness of the pier initially decreases, and is partially recovered at the end of the lifetime, when the decay of bending strength and the consequent loss of structural safety may become unacceptable. However, with respect to the importance level $\alpha=1.0$, the attainment of the minimum robustness and subsequent recovery is delayed from 20 to about 30 years. Moreover, a reversed trend is found for the larger values of the reference time t_0 . In fact, for $t_0=0$ the minimum robustness is achieved again for case (II). However, if robustness is computed for increasing values of the reference time t_0 , the susceptibility to damage decreases for case (II) and increases for case (I), and the latter becomes the worst scenario for $t_0=20$ years. These results demonstrate that bridges of

different importance should be designed by taking into account different target values of structural robustness. This can be achieved by means of the proposed importance factor.

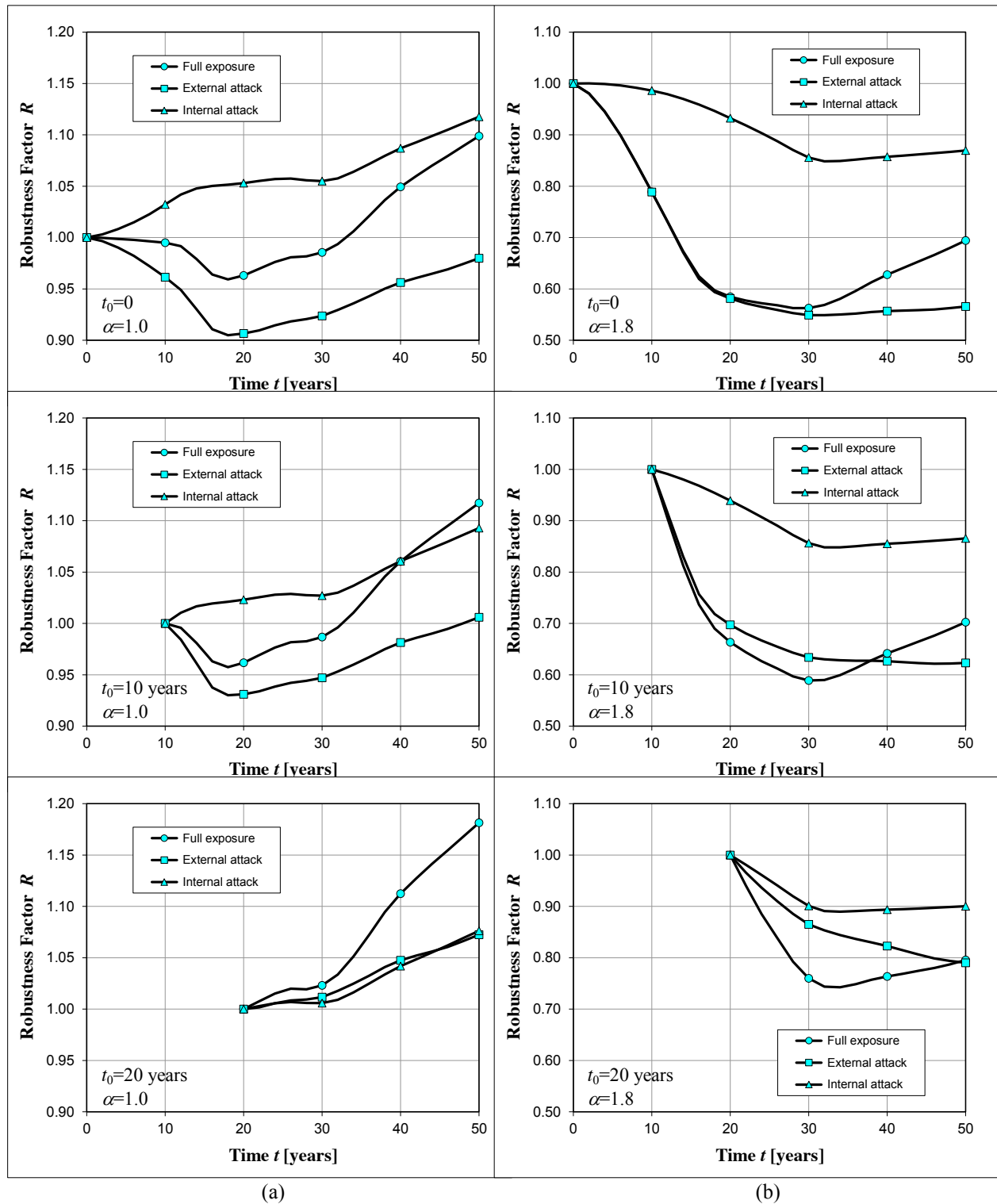


Figure 5. Time evolution of the time-variant robustness factor $R=R(t, t_0)$ associated with the three exposure scenarios for three reference time instants $t_0=0, 10$, and 20 years, and two values of the importance factor: (a) $\alpha=1.0$; (b) $\alpha=1.8$.

Finally, the effects of uncertainty on the lifetime structural robustness of the bridge pier is investigated for case (I) of full exposure with respect to the initial time $t_0=0$ by taking into account the importance of the bridge. The probabilistic model assumes as random variables the material strengths f_c and f_{sy} , the coordinates (y_p, z_p) of the nodal points $p=1,2,\dots$ which define the two-dimensional geometry of the RC cross-section, the coordinates

(y_m, z_m) and the diameter \varnothing_m of the steel bars $m=1,2,\dots$, the diffusivity coefficient D and the damage rates $q_c=(C_c\Delta t_c)^{-1}$ and $q_s=(C_s\Delta t_s)^{-1}$. These variables are assumed to be uncorrelated with the probabilistic distribution and statistical parameters listed in Table 2. Based on this model, a probabilistic analysis is performed by Monte Carlo simulation. A posteriori estimation on the goodness of the sample size is based on a monitoring of the mean and standard deviation of the robustness factor $R=R(t)$ for each time step over the lifetime.

Figure 6 shows the time evolution of the statistical parameters of the robustness factor $R=R(t)$ based on a sample of 3000 Monte Carlo realizations for $\alpha=1.0$ (Figure 6.a) and $\alpha=1.8$ (Figure 6.b). These results confirm the trend found from the deterministic analysis and show that the effects of uncertainty tend to increase over time periods when the susceptibility to damage increases and robustness decreases.

Table 2. Probability distributions and their parameters (*nom* = nominal value).

Random Variable ($t = 0$)	Distribution Type	Mean μ	Standard Deviation σ
Concrete strength, f_c	Lognormal	$f_{c,nom}$	5 MPa
Steel strength, f_{sy}	Lognormal	$f_{sy,nom}$	30 MPa
Coordinates of the nodal points, (y_i, z_i)	Normal	$(y_i, z_i)_{nom}$	5 mm
Coordinates of the steel bars, (y_m, z_m)	Normal	$(y_m, z_m)_{nom}$	5 mm
Diameter of the steel bars, \varnothing_m	Normal (*)	$\varnothing_{m,nom}$	$0.10\varnothing_{m,nom}$
Diffusivity coefficient, D	Normal (*)	D_{nom}	$0.10 D_{nom}$
Concrete damage rate, $q_c=(C_c\Delta t_c)^{-1}$	Normal (*)	$q_{c,nom}$	$0.30 q_{c,nom}$
Steel damage rate, $q_s=(C_s\Delta t_s)^{-1}$	Normal (*)	$q_{s,nom}$	$0.30 q_{s,nom}$

(*) Truncated distributions with non negative outcomes are adopted in the simulation process.

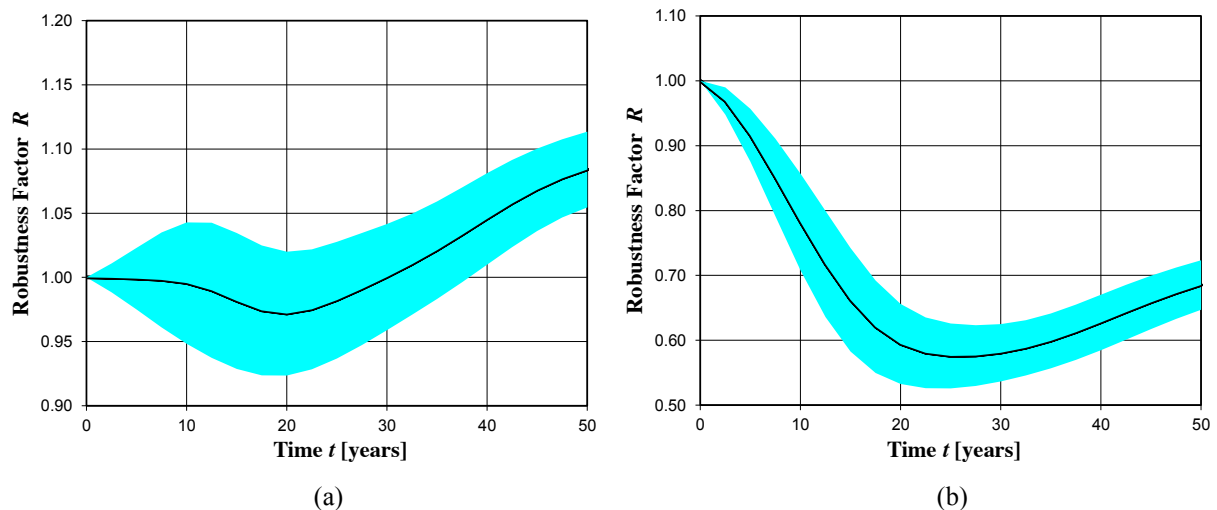


Figure 6. Time evolution of the statistical parameters of the time-variant robustness factor $R=R(t)$ computed with respect to the initial time $t_0=0$ for case (I) of full exposure, and for two values of the importance factor: (a) $\alpha=1.0$; (b) $\alpha=1.8$. The shaded region is bounded by the mean plus one standard deviation (upper bound) and the mean minus one standard deviation (lower bound).

CONCLUSIONS

The life-cycle robustness of deteriorating concrete structures has been investigated in deterministic and probabilistic terms. A robustness criterion proposed in previous works has been extended to consider the influence of the damage level on the time-variant robustness of existing concrete structures over the remaining structural lifetime. The proposed criterion allows to consider the relevance of the structure by means of an importance factor. Possible values of the importance factor have been provided for temporary structures, ordinary structures, and structures of strategic importance.

The proposed approach has been applied to the life-cycle robustness assessment of a RC bridge pier with box cross-section exposed to corrosion. Three exposure scenarios have been investigated: case (I) with full exposure of the cross-section, case (II) with external exposure only, and case (III) with internal exposure only. In terms of damage and strength deterioration the results showed that, as expected, case (I) is the worst scenario and case (II) is more critical than case (III). However, when the strength values are computed for the same amount of damage, different trends may arise. For bridges of ordinary importance robustness is maximum for case (III) with internal exposure, intermediate for case (I) with full exposure, and minimum for case (II) with external exposure. Moreover, if the assessment is made by considering the structure already damaged, robustness tends to be maximum in case (I). However, this tendency is reduced as the importance of the structure increases, and case (I) becomes the worst scenario also in terms of robustness for bridges of strategic importance.

These results are not intuitive and can be fully explained only through a proper time-variant measure of structural robustness able to account for the effects of damage under uncertainty. In fact, the probabilistic analysis confirmed the deterministic results and showed that the effects of uncertainty tend to increase over time periods when the susceptibility to damage increases and robustness decreases. This makes the robustness predictions less reliable when the system is not robust.

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